PROCEEDINGS OF THE ELEVENTH ASIAN REGIONAL CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING : SEOUL/ KOREA/ 16-20 AUGUST 1999

Soil Mechanics and Geotechnical Engineering
Eleventh Asian Regional Conference

Edited by
S.W.Hong, M.M.Kim, G.S.Yang, S.R.Lee, S.S.Chung, C.C.Ihm, H.T.Kim J.B.Park & B.S.Lee

Base grouting of wet process bored piles in Bangkok subsoil

Teparaksa W.
Chulalongkorn University, Thailand

Thasnanipan N., Anwar M. A.
Seafco Company Limited Bangkok, Thailand

A.A. BALKEMA / ROTTERDAM / BROOKFIELD / 1999

Eleventh Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, Hong et al. (eds)

Base grouting of wet process bored piles in Bangkok subsoils.

W. Teparaksa
Chulalongkorn University, Bangkok, Thailand.

N. Thasnanipan & M.A. Anwar
Seafco Company Limited. Bangkok, Thailand.

ABSTRACT: Base grouting techniques are widely used to rectify the soft toe problem of bored piles in Bangkok. Two different techniques, commonly known as flat jack and tube-a-manchette are used. This research study is mainly focused on the tube-a-manchette technique. Six piles were constructed with the provision of PVC-casing inside the shaft to allow the drilling through the pile bases in order to collect the soil samples below the pile tip. Piles were base grouted by varying the controlling parameters such as grout volumes, pressures and injection rates. Soil samples were collected from beneath and at some distance away from the pile tip, and analyzed. It is observed that grout does not penetrate into the surrounding sand of the pile toe even at high pressures. It partially replace and/or precompress the sediments present under the pile tip and most of the grout just rises up the soil/pile interface by replacing the bentonite cake. Recommendations are made to eliminate the soft toe problem more effectively.

1 INTRODUCTION

The method of constructing bored piles under bentonite suspension has become well established in Thailand. It is particularly used for large diameter piles, where powerful rigs can be used with rotary tools to bore through otherwise unstable water bearing strata. It is almost always used with a temporary casing to support the top most Bangkok Soft Clay (BSC), resulting in a hybrid of the casing and bentonite methods.

Construction problems are affecting integrity and performance of such piles. A pile without a sound toe and without a proper seat into the virgin ground should prove disastrous particularly if required to act in point bearing. Even otherwise, such piles could lead to higher orders of detrimental settlements. The known causes of ending up with a defective pile toe, are the loosened materials during boring operation and accumulation of sediments from the bentonite column at the base of the borehole. The technology of flowable concrete and tremie placing are well known to avoid this problem up to certain extent and it is already a standard procedure to use a cleaning bucket or an airlift or both to clear the borehole base of any soft or loose material.

Another solution was found in pressure grouting the base after the completion of the pile. This improves the condition of the soil and pre-stresses the pile. Base grouting is widely used in Thailand to rectify the soft toe problem.

2 BASE GROUTING IN BANGKOK

Teparaksa (1991-1992) reported the construction and performance of base-grouted bored piles in Bangkok subsoils. It was found that base grouting not only helps to improve the end bearing capacity but skin friction as well (ref: Table 1).

Table 1. Improvements made by base grouting (Teparaksa, 1994).

Layer of Pile tip	Increase in failure load (%)	Increase in skin friction (%)	Increase in end bearing (%)	Displacement at fully mobilized skin friction (% of Pile Dia.)	
				NGB	GB
Ist Sand	26-66	24-66	28-61	0.49 – 0.51	1.34-1.35
2nd Clay	27	51	1	0.37	0.74
2nd Sand	12-24	9-27	11-21	1.04	1.59-1.86

NGB: Non Grouted Base, GB: Grouted Base

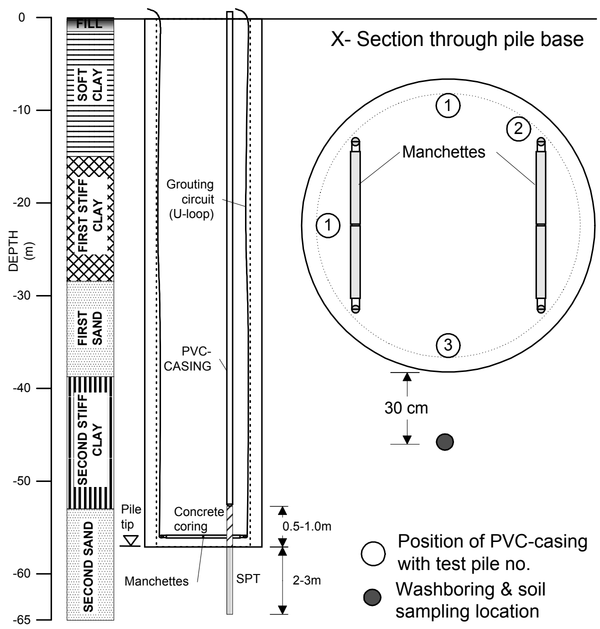
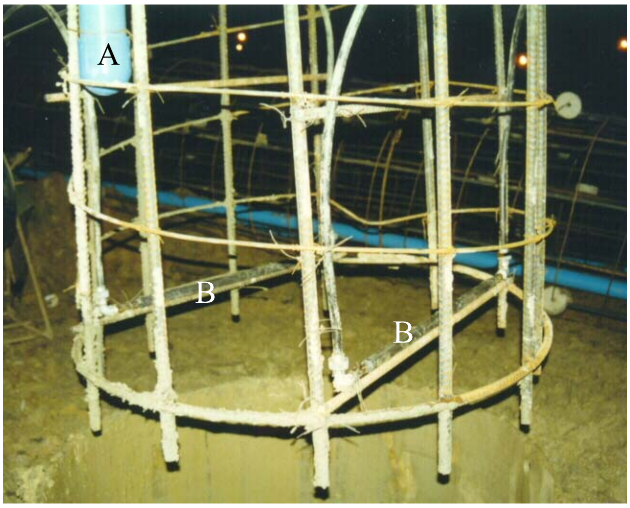


Figure 1. Schematic diagram of the positions of PVC casings in test piles with typical soil profile at the site.

Pile base grouting was started in Bangkok in 1985, during the construction of Rama IX cable stayed bridge (Morrison, 1987). Here 172 bored piles, 2.0 m dia. and 35 m long seated in the 1st sand were base grouted. Tube-a-manchette technique was used with grout volumes up to 4000 liters and maximum pressures of 60 bar. With the passage of time engineers started applying this technique for the foundations of expressways, high-rise buildings and other heavy structures. In the absence of standard guidelines, the criteria and the controlling parameters changed with time. In current practice, pressures ranging from 20 to 60 bar and grout volumes 500 to 1000 liters are normally applied with Injection Rate (IR) varying from 5 to 36 liters/min.

3 GROUTING PROCEDURE

Two grouting circuits in the form of U-loops with manchettes placed at a level of 5 to 10 cm above the pile tip arranged in a



A: Base of PVC-casing, B: Manchettes
Figure 2. PVC casing attached to the rebar-cage, before lowering in to the borehole.

symmetrical fashion are used (Figure 1). Normal practice is to use PE tubes above the manchettes running along the rebar cage to the top, but if specification require cross-hole sonic logging, same sonic logging tubes are used to connect the manchettes. Normal practice is to start grouting 24 hours after concreting when high pressure criteria is not to be strictly followed. This is especially suitable for congested piling plans where subsequent maneuvering of equipment can damage the circuit tubes. If high pressure is to be maintained and/or sonic logging is required, grouting is started after seven to ten days. First the pile toe is cracked with high-pressure water flow to flush and open the grouting circuit to make way for forth coming grout and then cement grouting is started. Grouting is stopped when the target grout volume or maximum required pressure is achieved.

4 TESTING METHODOLOGY

In order to investigate the mechanism of grout spreading beneath and in the surrounding soil of pile tip and the effect of different grouting parameters, an attempt was made to collect the soil samples from below the tip of the base grouted piles (Anwar, 1997). PVC-casings were placed in the prototype bored piles as shown in Figure 1 and 2, to allow for the passage of drilling tools down to the pile base. Three piles, 1.0 m in diameter and length in the range 50 to 55 m were constructed with the provision of 7.5 cm diameter PVC-casings at a site in Bangkok metropolitan area. Unfortunately, only one pile out of this group could be drilled because of the failure to lower the drilling tools inside the PVC-casings. For the second time another group of three piles having 1.5 m diameter and length 56 - 57 m were constructed. This time diameter of PVC-casing was increased to 10 cm. Positions of the PVC-casings were staggered with respect to the manchettes as depicted in Figure 1. All the three piles were base grouted following the parameters shown in Table 2.

Table 2. Grouting parameters of the test piles.

Test Pile	Grout Volume (Liters)	Injection Rate (Liters/min)	Maximum Pressure Achieved (bar)	Residual Pressure 2 min after closing the pump (bar)
TP-1	500	32 – 3*	26 – 16*	14
TP-2	545	36	36	3
TP-3	520	8	24	8

* *in two phases, refer Figure 4 for clarification.*

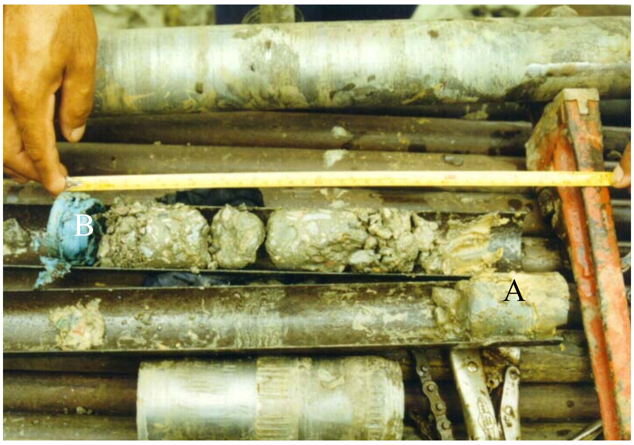
Portland cement Type I grout with w/c ratio equal to 0.55 was used. Volume of grout was measured from the number of pump strokes and also crosschecked against the volume consumed from the agitator tank. Nominal grout volume used for each pile was 540 liters but a slight difference shown in Table 2 in actual volume of grout measured might be due to the difference in IR used during grouting and at the time of calibration.

All the three piles were grouted with different IR varying from 3 to 36 liters/min. In each case, after injecting target grout volume, grout circuit valve was closed to lock the maximum pressure achieved. Residual pressures two minutes after closing the pump were also measured and are given in Table 2.

After 15 to 20 days of base grouting, diamond bit rotary drilling with double tube sampler was used to core through concrete present below the base of the PVC-casing. Right after reaching the pile tip, Standard Penetration Test (SPT) was started. Steel liners were used inside the split spoon sampler to collect soil samples. All the concrete and soil samples retrieved, carefully examined on site to determine the grout content present. Additionally, in order to determine the lateral spreading of grout into the surrounding soil of pile tip, additional boreholes were drilled 30 cm away from the pile periphery. Washboring was used down to approximately 1 m above the pile tip level and then SPT was employed to collect the samples.

5 CONCRETE CORES RECOVERED

From the concrete cores recovered no trace of grout was found except in case of test pile TP-1 where a 6.5 cm thick grout layer was recovered right below the pile tip as shown in Figure 3.



A: Grout core, B: PVC-casing bottom cap.
Figure 3. Grout layer of 6.5 cm thickness recovered from TP-1.

Below the grout core recovered no traces of grout were found in the sand samples recovered by SPT. Similarly for the rest of two test piles no traces of grout were seen in the soil samples recovered through SPT.

6 DEVELOPMENT OF GROUTING PRESSURE

Development of grout pressure with the increase in grout volume is shown in Figure 4. TP-1 was grouted with a high IR of 32 liters/min for the first 140 liters and then IR reduced to 3 liters/min for the rest of the grouting operation. It must be noted that as the IR is reduced, pressure dropped from 26 to 6 bar and then increased gradually to 16 bar at the end of the grouting. Two minutes after closing the pump, pressure dropped to 14 bars. TP-2 was grouted at a constant and very high IR of 36 liters/min, which corresponds to the maximum available speed of the grout pump. It must also be noted that pressure development in this case was very quick as it jumped to 15 bars within first few strokes of grout and then rose to 36 bar at about 200 liters. No further increase in pressure was observed till the target volume of grout had been pumped. Pressure abruptly dropped to about 3 bar two minutes after closing the pump. TP-3 was grouted with a constant IR of 8 liters/min. Development of pressure in this case was quite gradual and it approached maximum pressure of 24 bar and dropped to 8 bar 2 min after closing the pump.

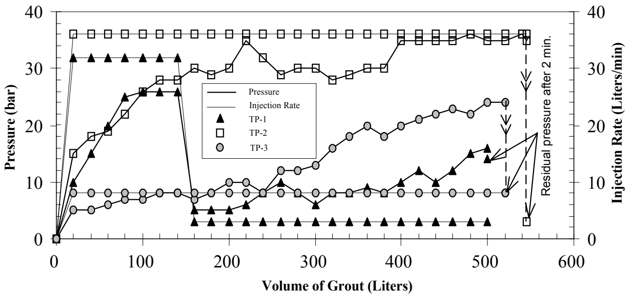


Figure 4. Curves showing gradual increase in grout pressure.

7 GROUT INJECTION RATE

7.1 *High Injection Rate*

It has been observed that IR is a key factor in the process of base grouting. If IR is kept very high, grout pressure rises abruptly but can not be smoothly maintained. A clear fluctuation in pressure is observed during grouting with high IR. Although the apparent maximum pressure indicated by the dial gauge is high at the initial stages but can not be increased further. It seems that at high IR, hydraulic fracturing of the surrounding soil took place which is also evidenced by sharp fluctuations in pressure during grouting Similar phenomenon have been observed by Yeats, 1989. It must also be noted that maximum pressure achieved in this case dissipates very quickly as the grouting operation is terminated. This means that pressure developed in this case mainly consists of the circuit resistance and is not the true pressure induced at the pile/soil interface. Since pressure ceases to increase beyond 400 liters it seems that no further increase in pressure is possible even if large volumes of grout have been injected.

7.2 *Low Injection Rates*

If IR is kept low, development in pressure took place quite gradually and the pressure fluctuation is minimal. It must also be noted that the pressures developement by keeping IR low is quite steady and can also be maintained for certain duration of time, as shown in Figure 4 for TP-1 where a residual pressure of 14 bars has been observed two minutes after closing the pump with a total drop of only 2 bar. It must also be noted that pressure keeps on increasing gradually and higher pressures than that of TP-2 can be achieved with more grout volumes.

8 PENETRATION OF GROUT INTO THE SAND.

From the samples collected, no traces of grout have been observed in the sand present below or surrounding the pile tip. Grain size distribution curves of sand samples collected are shown in Figure 5. It can be seen that the second sand layer falls in the range of medium to fine sand range. Grain size distribution for different types of available cements is also shown in Figure 5.

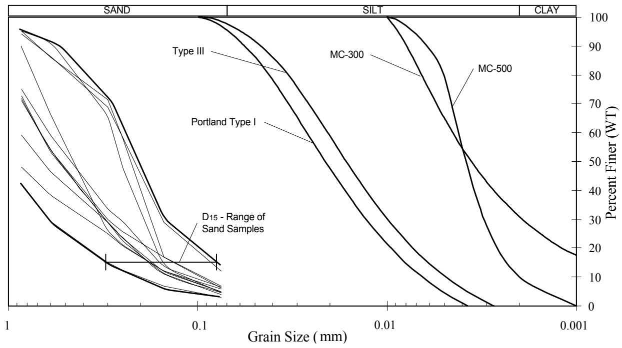


Figure 5. Grain size distribution curves of samples collected from Second Sand Layer around the pile tip.

In order to assess the possibility of permeation grouting of the 2nd sand layer present around the pile tip, Groutability Ratio (GR) of the sand layers was determined. GR is a useful parameter for checking the applicability of grout for use in sand and is given by the formula as follows (Mitchell, 1970):

$$GR = D_{15}/D_{95} \tag{1.1}$$

Where D_{15} = The particle diameter of the soil to be grouted 15% of which is finer by weight; and D_{95} = the particle diameter of the grout 95% of which is finer by weight.

Weaver (1991) summarizes the possibility of grouting a soil for GR ranges as:

GR > 24, usually;
GR < 19, not likely; and
GR < 11, not possible.

GR range for the 2nd sand layer is found to be 1.2 to 4 for the Portland cement type I which means that its not possible for the cement grout to permeate into the sand layers. GR using microfine cements MC-300 (Henn, 1996) shown in Figure 5 comes out to be 12 and 30, so the silty sand layer encountered can be permeation grouted using microfine cements.

9 INSTALLATION EFFECTS

SPT-N values obtained right below the pile tips and 30 cm away from the pile are compared with the values from nearest borehole (10 – 20 m) performed at the time of soil investigation in Figure 6. It is quite clear that the density of the sand surrounding the tip have been increased appreciably, especially, below the tip of the piles.

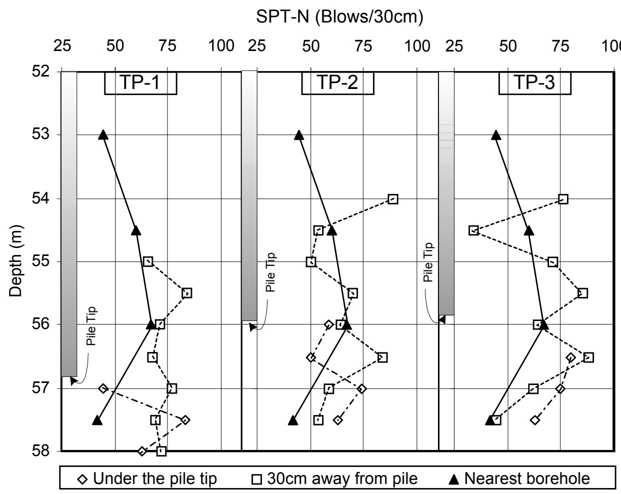


Figure 6. Comparison of SPT-N values around the grouted pile tip and the nearest borehole from soil investigation.

10 MANCHETTE BREAKING MODELS

Thickness of concrete cover present below the manchette is very important and determines the flow path of the grout injected. If concrete cover present below the manchette is thicker than the side cover to the outlet points on manchette, it can be suspected that grout directly percolates toward the pile shaft by bursting the side cover. In order to monitor the bursting mechanism of manchettes, 9 model piles 1.2 m diameter and 2.0 m deep were constructed by varying manchette levels above the pile tip (Thasnanipan et al, 1998). Model piles were water burst in a similar way comparable to the normal bored piles. Colored grout was injected to differentiate it from normal pile concrete. Suitable time was given to allow for the grout setting before these piles were exhumed to determine the flow pattern of the injected grout. Some of the unearthed model piles are shown in Figure 7 and 8.

Figure 7. Typical model pile used to determine manchette breaking mechanism.

From the analysis of the model piles, it was revealed that: a) full pile base area coverage by grout can be achieved if manchette level is less than 20 cm above the pile tip; b) a partial base area coverage was found up to 30 cm and if; c) manchette level is more than 40 cm above the tip, grout directly percolates towards the pile shaft by bursting the side concrete cover. Presence of grout on the vertical pile/soil interface at the shaft was also confirmed in all cases. Although a direct comparison of grout flow in model piles and prototype piles can not be made due to the difference in overburden pressures but the upward flow of grout along the pile shaft can be the most likely phenomenon in base grouting. Which, subsequently act as a '*rock socket*' and significantly increases the pile stiffness (Francescon, 1992).

11 CONCLUSIONS

From the study carried out following conclusions can be drawn.

1. Most of the grout injected during base grouting rises up along the pile/soil interface, and does not permeate in to the surrounding sand layer to any degree. So the improvement made by base grouting is mainly contributed by increase in skin friction in the form of rock-socket effect which increases the pile stiffness significantly. Since the increase in skin friction depends only upon the volume of grout injected, pressures achieved during grouting are of negligible importance in this regard. Partial replacement and/or precompression of the sediments below the pile tip is possible by following low IR hence, smoothly increasing and maintaining the pressures achieved, if it is desired so.
2. IR is the key factor in the base grouting process, especially if, higher pressures are to be achieved and maintained.

A: colored grout zone

Figure 8. Model pile showing the grouted zone below the tip.

   Low IR of the order of 3–5 liters/min are recommended for base grouted piles in silty sand layers of Bangkok.
3. Significant improvement in the density of sand around the pile shaft near the base and especially, below the grouted pile tip has been observed. Although this is also contributed by increased overburden pressure due to concrete, base grouting helps to improve the soil around the pile tip, disturbed during the drilling process.
4. Position of manchettes above the pile base is very important to effectively grout the base of the pile. Care must be exercised to place the manchettes as close to the pile tip as possible but if due to some unforeseen reasons manchettes are lifted up grout can be conjectured to percolate towards the pile base if the concrete cover thickness below the manchettes does not exceed 30 cm.

REFERENCES

Anwar, M.A. 1997, base grouting to improve the performance of wet process bored piles in Bangkok sub-soils, M. Engg. Thesis, AIT, Bangkok, Thailand.

Francescon, M. & Solera, S.A. 1992, Base grouting to improve performance of piles bored under bentonite in the Thanet Sands, Proc. of the Conf. on Grouting in the ground, ICE, London.

Henn, R.W. 1996, Practical guide to grouting of underground structures, ASCE publication, NY.

Mitchell, J.K. 1970, In-place treatment of foundation soils, Journal of the Soil Mechanics and Foundation Division, ASCE, 96(1), 73-110.

Morrison, I.M. 1987, Bored piled foundation for Chao Phya river crossing at Wat Sai, Bangkok, 9th SEAGS Conf. Bangkok, pp. 6-207 to 6-218.

Teparaksa, W. 1991, Performance of base-grouted bored piles in Bangkok subsoils, First Young Asian Geotechnical Engineers Conference, AIT, Bangkok, Thailand, pp.117-128.

Teparaksa, W. 1992, Behavior of base-grouted bored piles in Bangkok subsoils, Piling Conference, European Practice and Worldwide Trends, Institution of Civil Engineers, London, pp.296-301.

Teparaksa, W. 1994, Newly developed toe-grouted bored piles in soft Bangkok clay: performance and behavior, Proc. of the Intl. Conf. on Design and Construction of Deep Foundation, FHWA, Orlando, USA, pp.1337-1351.

Thasnanipan, N. et al, 1998, Design, Construction and behavior of bored cast in-situ concrete piles in Bangkok subsoil, Proc. 4th Intl. Conf. on Case Histories in Geotechnical Engineering, St. Louis, Missouri, USA, paper no.1.20.

Thasnanipan, N. & Umesh, S. 1998, Pile base cracking models with different manchette levels from the pile tip, SEAFCO internal report.

Weaver, K.D. 1991, Dam Foundation Grouting, American Society of Civil Engineers, NY.

Yeats, J.A. & O'Riordan, N.J. 1989, The design and construction of large diameter base grouted piles in Thanet Sand at Blackwall Yard, London, Proc. Intl. Conf. on Piling and Deep Foundations, London, Vol. 1, pp. 455-461, Balkema.